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# On Structural Modelling for Seismic Analysis with Hydrodynamic Interaction

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## Introduction

The hydrodynamic interaction of the structural components of a dam is investigated for earthquake induced effects. Structural deformability and water compressibility are taken into account. Results are compared to those obtained by traditional analysis. A practical approach is developed for adapting available computer software to the specific requirements of the existing structure. The modelling of the structural elements is incorporated in the analysis of the global structural behaviour under conditions of maximum credible earthquake.

In recent years the seismic analysis of dams has focused renewed attention on the significance of hydrodynamic interaction. Earthquake predictions have grown increasingly ominous while structural performance during earthquakes consistently baffles predictions. A number of dams have been built in earthquake — prone areas. With the refinement of seismic risk assessment and the added sofistication of computational means for dynamic linear and non-linear analysis, it has become feasible to reanalyse such sensitive structures applying the state of the art techniques.

Such is the case of the Tarbela Dam in Pakistan, designed by TAMS Engineers, Architects and Planners of New York during the 1960's and 1970's. With landfill embankments of 145 924 000 $m^3$ and a capacity of 13 692 billion $m^3$, Tarbela is considered the world's largest earthfill dam.

Recognizing the substantial progress in seismic prediction and structural analysis since it's original design, TAMS conducted a dynamic reanalysis of Tarbela during the early 1980's [1]. The present article addresses one aspect of this reanalysis. It deals with the study of an auxilliary concrete spillway with a steel gate controlling the water level.

### Description of the system

The considered system is comprised of a soil foundation and a concrete monolith which supports a steel gate, anchored to a trunnion beam. Along their upstream face the monolith and the gate are in contact with water which, for this analysis is assumed at high level.

Under these conditions, an earthquake acting parallel to flow is anticipated. A particularly sensitive link forms at the gate anchorage, where both the mass of the system and it's acceleration can have a compounded damaging effect. The complexity of the system is such that it becomes necessary to approach the problem in several consequtive steps.

### Objective

A three-dimensional finite element model of the system is shown in Fig. 1 [1]. The purpose of the analysis is to evaluate the seismically induced forces acting on the gate anchorage. While the model shown in Fig. 1 already incorporates a number of significant assumptions beyond the scope of this paper, one primary concern — the modelling of the water action on the upstream face of the gate and the concrete — remains unresolved.

### Modelling for Hydrodynamic Interaction

According to the classical methods [2, 3], discrete water masses are determined and applied to the system which, for this purpose is regarded as rigid. In [4] it is argued that ignoring the deformability of the structure and in effect neglecting the hydrodynamic interaction between water and concrete is excessively simplistic and can lead to serious underestimates of the stresses induced. The argument is even better substantiated in the case of a steel gate which is lighter and more flexible. The authors of [4] have developed the program EADHI [5] for computation of hydrodynamically induced stresses in gravity dams. The program adopts a substructure method and solves the interaction problem both with and without considering water compressibility. The structure is modelled by plane strain finite elements. The fluid is represented as a continuum. The solution is obtained in the frequency domain. EADHI is computationally efficient and convenient to use but has certain limitations. Having been designed primarily (if not exclusively) for concrete monoliths it can only work with straight upstream faces, normal to the water flow and with plane strain finite elements.

In the present case EADHI has to accomodate the complex geometry of the steel gate. As demonstrated by the gate model of Fig. 2 — A, B, C, this is best described as a space frame with a convex face toward the water.

At this point it is recalled that EADHI solves the equations of motion in terms of complex frequency response functions. Thus it can be argued that a model with a geometry admissible to EADHI and with the same general properties, most important among which should be the fundamental natural frequencies, as the gate model, can be substituted for it.

The search for such a substitute model is guided by the usual principles of simulation technique, i. e. that simplicity of the model is desirable, unnecessary parameters are to be avoided, parameter sensitivity should be checked and all significant aspects of the original's behaviour should be duplicated as closely as possible. An example of the substitute model fairly similar to the one used for the final report is shown in Fig. 2 — D, E, F. The properties, maintained identical in both the gate model and it's substitute are:

A. Total mass,

B. Overall height,

C. Added water masses,

D. Support conditions,

E. Axial stiffness of the members and the finite elements representing the gate arms (the mass of these elements and members is already identical, according to A),

F. Flexural stiffness of the vertical layers of finite elements and the gate vertical diaphragms is kept as close as possible. This is deemed sufficient for the excitation parallel to the flow.

Three vertical layers of finite elements are used to simulate the four vertical diaphragms of the gate model, which, in turn already incorporate the stiffness of the continuous gate surface.

| No | Descr. | $E^* \ 10^6$ [kg/cm²] Static | $E^* \ 10^6$ [kg/cm²] Dynamic | $W$ [kg/cm²] Static | $W$ [kg/cm²] Dynamic | $\mu$ |
|---|---|---|---|---|---|---|
| 1 | *Pier* | .2461 | .3076 | .0024 | .0024 | .15 |
| 2 | *Ogee weir* | .2250 | .2812 | .0024 | .0024 | .15 |
| 3 | *Soil FDN* | .0211 | .0211 | .0019 | O | .25 |
| 4 | *Steel gate* | .20389 | .20389 | .0078 | .0078 | .27 |

MATERIALS

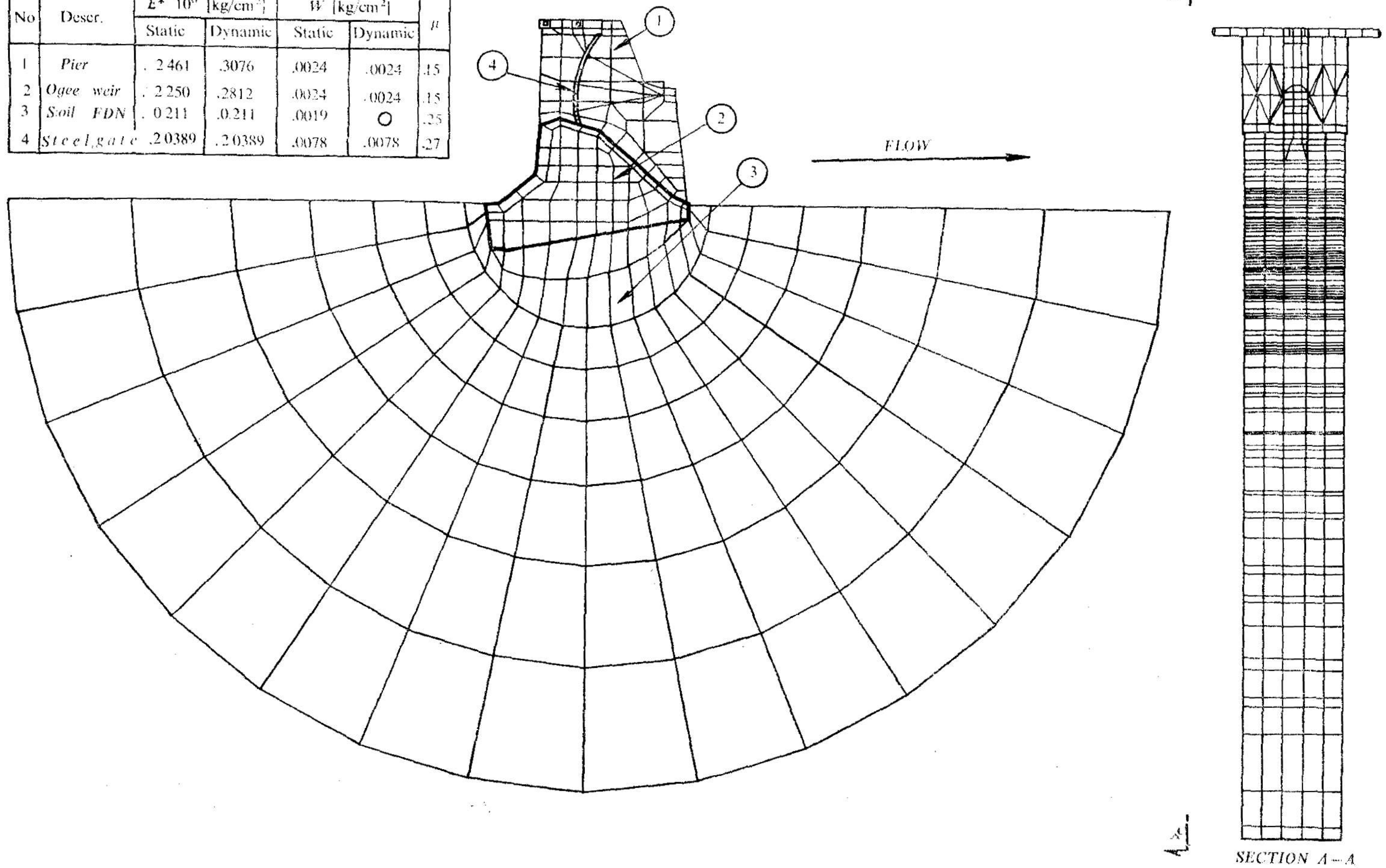
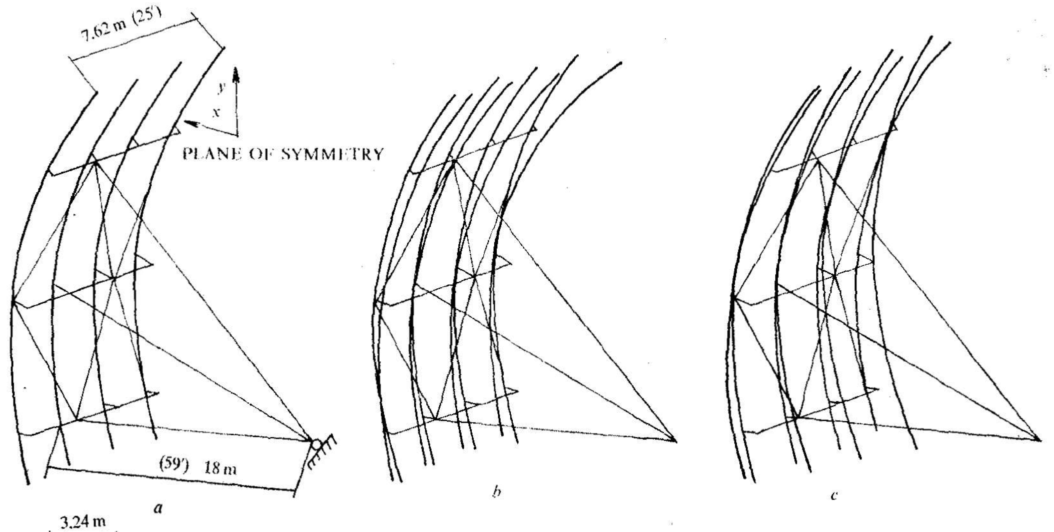
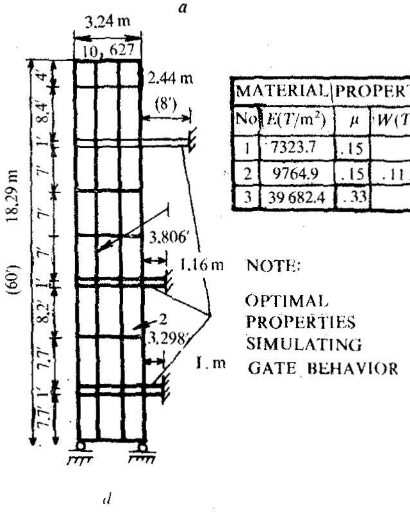


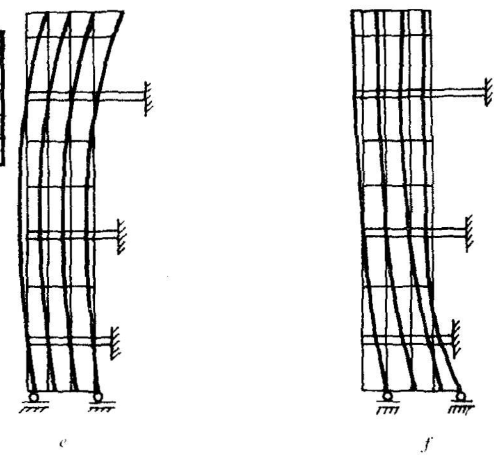

Fig. 1. Spillway Gate Anchorages for Earthquake Parallel to Flow a, b, c, d, e, f




| NATURAL FREQUENCIES OF GATE MODEL (Hz) | | |
|---|---|---|
| | WITHOUT WATER MASSES | WITH WATER MASSES (ZANGAR) |
| 1 | 11.38 | 3.66 |
| 2 | 14.27 | 4.11 |
| 3 | 18. | 5.57 |
| 4 | 18.61 | 5.62 |
| 5 | 19.41 | |
| 6 | 19.71 | |
| 7 | 23.42 | |
| 8 | 24- | |




| MATERIAL PROPERTIES | | | |
|---|---|---|---|
| No | $E(T/m^2)$ | $\mu$ | $W(T/m^3)$ |
| 1 | 7323.7 | .15 | |
| 2 | 9764.9 | .15 | .11533 |
| 3 | 39682.4 | .33 | |




| NATURAL FREQUENCIES OF SIMPLIFIED MODEL (Hz) | | |
|---|---|---|
| | WITHOUT WATER MASSES | WITH WATER MASSES (ZANGAR) |
| 1 | 11.32 | 3.6 |
| 2 | 13.27 | 3.66 |
| 3 | 20.89 | 4.24 |
| 4 | 25.49 | 4.42 |
| 5 | 31.27 | 5.53 |

Fig. 2. Spillway Gate Models: a) model of half spillway gate; b) mode with trequency w=11,38 Hz (no water mass); c) mode with frequency w=3,66 Hz (water masses-zangar); d) simplified gate model for interaction solution; e) mode with frequency w=11,32 Hz (no water mass); f) mode with trequency w=3,66 Hz (water masses-zangar)

The adjustment of the substitute model's behaviour is accomplished by varying the Poisson ratio, the Young modulus, the density and shape of the finite elements until a combination best suited to the task is achieved.

As illustrated in Fig. 2, a satisfactory degree of similarity can be attained between the lower natural frequencies of the compared models with and without the added action of water masses. Also comparable are the corresponding mode shapes. After several modifications beyond the one shown in Fig. 2 — D, E, F, results seemed convergent without undue sensitivity to parameter variation and were adopted for further use.

The next step is to examine the forces in the elements of the substitute model, representing the gate arms. A comparison of these forces without and with hydrodynamic interaction, including water compressibility in one case, is shown on Table 1. It is apparent that hydrodynamic interaction results in significant changes. Furthermore, the usually neglected water compressibility contributes a demonstrably adverse effect in this case and must be taken into account.

### Analysis of the Global Model

The above results are incorporated in the three-dimensional model of Fig. 1 in the following manner. Water masses are obtained for the geometry of the global model according to Zangar [3]. Subsequently, these values are multiplied by a ratio of the forces, determined with and without hydrodynamic interaction for the substitute model. These ratios are listed in column 5 of Table 1 and are equal to the corresponding values in column 3, devided by those in column 2.

The global model with the added modified water masses is subjected to a spectral analysis. The Seed — Idriss accelerogram is used, scaled to the estimated maximum credible earthquake magnitude of 0.5 *g*, with 5 % damping.

It should be noted that, alternatively at this stage a time-history analysis could be used. It is known that on occasion time — histories tend to yield higher maximum stress values than the corresponding spectra. On the other hand, time-histories are much more time-consuming and, hence expensive. Consequently in cases when the sole concern are the maximum stresses at predefined locations, the time and cost efficiency of the spectral analysis makes it the preferable choice.

The results of the above analysis for the space frame members representing the gate arms are listed on Table 2. While it is reasoned that overstresses in the highly flexible gate arms could be easily remedied without grave distress to the operation of the entire facility, damage to the anchorage should be avoided, by preventive measures if necessary.

### Conclusions

The forces quoted on Table 2 exceed those obtained by hydrostatic analysis (not quoted here) by a factor of approximately 5. This is largely due to the magnification of the ground acceleration at the level of the trunnion beam supporting the gate. Of critical importance to the above is the modelling of the soil under the monolith. No less important, however, is the contribution of the hydrodynamic interaction. Both the structural deformability and the water compressibility have a demonstrable effect on the outcome of the analysis.

In the absence of solutions and algorithms, specifically designed for structures of complex configuration, it is shown that models with simplified geometry can be successfully substituted.

Table 1

**Maximum Arm Forces in Gate Substitute Model for Alternative Assumptions (Tons per Gate Arm, per half gate):**

| Location | Type of Assumption | | | Ratio |
|---|---|---|---|---|
| | Added Masses (Westergaard), No interaction | Results obtained by EADHI | | 3/2 |
| | | Interaction, Compressibility | Interaction, No compressibility | |
| 1 | 2 | 3 | 4 | 5 |
| Top Arm | 23.59 | 26.40 | 20.96 | 1.119 |
| Middle Arm | 23.31 | 41.41 | 29.89 | 1.776 |
| Bottom Arm | 4.23 | 34.29 | 26.17 | 0.775 |
| Total | 9.13 | 102.1 | 77.02 | |

Table 2

**Arm Forces for Spillway Gates Obtained by Spectral Analysis of Global Model (Fig. 1), Seed — Idriss, 0.5 *g* (Tons per Gate Arm, per half gate)**

| Arm | Force | Horizontal Component | Vertical Component |
|---|---|---|---|
| Top | 1762.2 | 1545.4 | —836.4 |
| Middle | 2992.8 | 2980.1 | —374.2 |
| Bottom | 2987.4 | 2911.6 | 621.4 |
| Total | | 7437.1 | —589.2 |

## Acknowledgement

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## References

1. Report on Dynamic Reanalysis of Existing Concr. Structures at Tarbela, Draft — June 1983, TAMS Engineers, Architects and Planners, New York.
2. Westergaard H. M. Water Pressure on Dams During Earthquakes Transactions ASCE, vol. 98, 1933, pp. 418-433.
3. Zangar C. N. Hydrodynamic Pressures on Dams Due to Horizontal Earthquake Effects — Engineering Monograph No. 11, US Bureau of Reclamations, 1952.
4. Chakrabarti P., A. K. Chopra. Earthquake Analysis of Gravity Dams Including Hydrodynamic Interaction — Earthquake. Engineering, 1984.
5. Chakrabarti P., A. K. Chopra. A Computer Program for Earthquake Analysis of Gravity Dams Including Hydrodynamic Interaction — Report No. EERC 73, 7, U. C., Berkeley, CA.

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